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Geotechnical Design Memorandum
Excavation and Removal of Soil
Port of Portland Terminal 1 South
Portland, Oregon

Prepared for
Port of Portland

April 23, 2002
15230-02

Prepared by
Hart Crowser, Inc.

REGISTERED PROFESSIONAL ENGINEER 14,431
OREGON JULY 26, 1989
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POPT1S601007

CONTENTS

FIGURES

APPENDIX

GEOTECHNICAL DESIGN MEMORANDUM
EXCAVATION AND REMOVAL OF SOIL
PORT OF PORTLAND TERMINAL 1 SOUTH
PORTLAND, OREGON

1.0 INTRODUCTION

This report presents Hart Crowser's geotechnical engineering evaluation and recommendations for soil excavation, removal and replacement for The Port of Portland, Terminal 1 South (T1S), Portland, Oregon. The project site, T1S Site, is located at 2100 NW Front Avenue in Portland, Oregon (Figure 1). The site consists of approximately 21 acres that are almost completely paved with asphalt or concrete or covered by buildings (Figure 2). Two primary structures, designated as Warehouse No. 2 and House No. 104, are currently located at the T1S Site. An extensive dock structure is present over submerged lands at Berths 104, 105, and 106.

The project will include a number of localized environmental excavations within the bounds of T1S. Hart Crowser's geotechnical focus for this project was one of assessing practical approaches to excavations located adjacent to public rights-of-way and buildings. After this introduction, the report presents the following:

- Summary;
- Project understanding;
- Subsurface conditions;
- Engineering conclusions; and
- Appendix addressing field explorations and laboratory data.

1.1 Purpose of Work: Provide Geotechnical Engineering Recommendations

The purpose of our work was to provide geotechnical engineering recommendations for design and construction of the proposed work. Our recommendations include:

- Excavations adjacent to existing roads, sidewalks, utilities, and buildings;
- Grading, filling, and compaction recommendations for backfill; and
- Other pertinent geotechnical design criteria and construction considerations.

1.2 *Scope of Work: Soil Explorations, Engineering Analyses, and Report Preparation*

Our scope of work for this project included:

- A review of general geologic literature and previous geotechnical reports in the project vicinity;
- Surficial reconnaissance;
- Subsurface explorations;
- Laboratory testing;
- Geotechnical engineering analyses; and
- Preparation of this report.

1.3 *Limitations of Our Work*

Hart Crowser performed this work for the exclusive use of the Port of Portland, their clients, and agents for specific geotechnically related applications to this project. This work was conducted in accordance with generally accepted professional practices in the same or similar localities, related to the nature of the work accomplished at the time the services were performed. No other warranty, express or implied, is made.

2.0 SUMMARY

Following is a summary of the findings in this report. Please refer to the full report for all of the assumptions and details regarding our findings.

2.1 *Subsurface Conditions*

Asphalt Concrete Pavement. T1S is presently paved over the majority of the project site. Asphalt concrete thickness observed in the field varies widely. Asphalt cores obtained during site exploration varied in thickness from 2.75 to 9 inches in thickness.

Fill. Fill of varying constituents and soil matrix was encountered in all four exploratory borings. The surface fill horizon was only fully penetrated in boring B-1. The contact between fill and native coarse sand and gravels underlying the site within B-1 was encountered at approximately 32 feet below ground surface (bgs).

Native Coarse Sands and Gravels. A dense, wet, gray, coarse sand with lenses of gravel was encountered within B-1 at depths below approximately 32 feet.

Groundwater. The static groundwater table was encountered at varying depths below the ground surface. Within boring B-1 groundwater was encountered at approximately 26.5 feet bgs at the time of drilling. Groundwater was encountered at shallower depths within borings B-3 and B-4 with depth at the time of drilling being respectively 18.5 feet and 16.5 feet. Groundwater depths are expected to rise and fall seasonally and will typically be encountered at depths corresponding to water levels in the adjacent Willamette River.

2.2 Site Preparation and Excavations

It is strongly recommended that the work be conducted during warm, dry months.

Excavations and Cuts. Excavations and cuts can often result in settlement or loss of support of the surrounding ground surface. These settlements may be sufficient to cause damage or distress to buildings, retaining walls, utilities, services, or other structures located near the excavation, which are founded in areas above the base of the excavation. Recommendations for setbacks, shoring, underpinning, and temporary construction slopes have been provided in the body of this report.

Compaction Standards. Recommended compaction specifications should be based upon ASTM D 1557 (Modified Proctor Test) or AASHTO T-180. It is our understanding that excavation backfill will subsequently be removed during the final site development. Therefore, backfill soils should be compacted to at least 90 percent of the material's maximum dry density.

3.0 PROJECT UNDERSTANDING AND SITE DESCRIPTION

3.1 Site Location

The T1S Site is located at 2100 NW Front Avenue along the Willamette River in Portland, Oregon (Figure 1). The site consists of approximately 21 acres located northwest of Interstate 405 (Fremont Bridge), northeast of NW Front Avenue, southeast of Slip No. 2, and southwest of the Willamette River (Figures 1 and 2).

3.2 Site Description

Two primary structures, designated as Warehouse No. 2 and House No. 104, are currently located at the T1S Site. Additionally, an extensive dock structure is present adjacent to the T1S Site over submerged land at Berths 104, 105, and 106.

The topography at the T1S Site is generally level at an elevation of approximately 30 feet above mean sea level (msl). The site is paved with asphalt or concrete with no vegetation and little bare ground present.

4.0 GEOLOGIC SETTING

Site geology consists of near surface man-made fills mantling Quaternary Alluvium deposits. Based upon review of The Department of Geology and Mineral Industries, Open File 0-90-2, "Earthquake-Hazard Geology Maps of the Portland Metropolitan Area, Oregon", (1990) by Ian Madin, the above geologic units are defined as Qaf, and Qal.

The Qaf unit (Artificial Fill) is widespread in developed areas along the flood plains of the Columbia River and Willamette River. The material often consists of uncontrolled dredged river sands, although larger material such as concrete debris, asphalt debris, or deleterious material is common to this fill unit. Placement of this material was often conducted hydraulically resulting in fills that are normally consolidated. Long-term settlements within this fill unit are common as the material consolidates under self weight.

Underlying the above-described Qaf Unit is a Quaternary aged alluvial flood deposit material (Qal). The Qal unit typically consists of silts, sand, and gravel sized material deposited along the Columbia and Willamette River Valleys during multiple catastrophic flooding events some 10,000 years ago. Sand and gravel deposits can be loose to dense in character, with varying amount of fine-grained constituents. The Qaf within the project vicinity is typically underlain by the late Pliocene aged Troutdale Formation at depths of 50 to 100 feet below the existing ground surface elevation.

5.0 SUBSURFACE CONDITIONS

Numerous subsurface explorations have been conducted on this site in the past. Several additional geotechnical borings were advanced into areas of the proposed soil excavation and removal areas to supplement this past field data. Supplementary field explorations for this project were conducted on March 25, 2002. This supplemental exploration consisted of advancing several hollow stem augured borings to depths of 21 to 41 feet bgs.

The approximate locations of these borings are shown on the Site Plan (Figure 2) and the logs for the borings are provided in Appendix A of this report. The locations of borings B-1 and B-2 were paced off in the field from prominent surface features, and the locations indicated on Figure 2 should be considered approximate. The locations of borings B-3 and B-4 were located by surveyors from KPFF Consulting Engineers.

Soil conditions encountered within borings were logged in the field by a representative of Hart Crowser's geotechnical engineering staff. Logs of all subsurface explorations have been included in Appendix A of this report. The attached boring logs describe soils and various engineering properties of soils encountered during exploration. Descriptions are based upon field classification of soil samples.

It should be emphasized that our exploration revealed subsurface conditions only at discrete locations on the project site and that actual conditions could vary at other locations. Furthermore, the nature and extent of any such variations would not become evident until construction activities have begun. If significant variations are observed at that time, we may need to modify our conclusions and recommendations to reflect actual conditions. For ease of outside interpretation, subsurface conditions have been generalized into the following major categories.

Asphalt Concrete Pavement. Asphalt concrete thickness observed in the field varied widely. Asphalt cores obtained during site exploration varied in thickness from 2.75 to 9 inches in thickness. The borings (B-1 and B-2) advanced along the northern proposed excavation area indicated asphalt thicknesses that varied between 5.5 to 9.0 inches. The two southern borings were advanced through approximately 2.75 to 3.5 inches of asphalt pavement. Base rock thickness within all boring was limited to less than 4 inches of gravel.

Fill. The site has been the subject of numerous past filling operations. The fill observed in borings B1 through B-4 was somewhat variable in content, with material ranging from soft sandy silts to loose to medium dense, medium grained sand with some to a little silt. Lenses of debris were also noted during soil sampling. Debris consisted of buried lumber as well as trace amounts of other organics (roots, wood). Larger diameter debris material is visible along the northern slip. The embankment in this vicinity is mantled with large pieces of asphalt, concrete and boulders. Although not encountered in borings B-1 through B-4, it is possible that such material may become evident during site excavation work.

In general the compactive effort employed during fill placement, based upon blow count data, was probably minimal. Another indication of fill compactive effort is the large variability in asphalt thickness. We presume that settlement and pot-holing resultant from loose fill have been progressively backfilled over time and patched with additional lifts of asphalt concrete.

The surface fill horizon was only fully penetrated in boring B-1. The contact between fill and native coarse sand and gravels underlying the site within B-1 was encountered at approximately 32 feet bgs.

Native Coarse Sands and Gravels. A dense, wet, gray, coarse sand with lenses of gravel was encountered within B-1 at depths below approximately 32 feet.

Groundwater. The static groundwater table was encountered at varying depths below the ground surface. Within boring B-1 groundwater was encountered at approximately 26.5 feet bgs at the time of drilling. Groundwater was encountered at shallower depths within borings B-3 and B-4 with depth at the time of drilling being respectively 18.5 feet and 16.5 feet. Groundwater depths are expected to rise and fall seasonally and will typically be encountered at depths corresponding to water levels in the adjacent Willamette River.

6.0 GEOTECHNICAL ENGINEERING CONCLUSIONS AND RECOMMENDATIONS

Our recommendations are based on our current understanding of the project. If the nature or location of the planned construction changes, Hart Crowser should be contacted so that we can confirm or revise our recommendations.

6.1 Excavations, Temporary Cuts, and Shoring

Subsurface conditions encountered during the site investigation indicate that precautions in excavations will be required due to the potential for caving/sloughing within fill soils underlying the site. Any excavations deeper than 4 feet should be sloped or shored in accordance with OSHA regulations. Dewatering of perched groundwater and/or rainwater within excavations may also be required.

Excavations and cuts can often result in settlement or loss of support of the surrounding ground surface. These settlements may be sufficient to cause damage or distress to buildings, retaining walls, utilities, services, or other structures located near the excavation, which are founded in areas above the base of the excavation.

Temporary excavation slopes may be suitable in areas where adjacent improvements are not located within a horizontal distance equal to the depth of the excavation (measured from the top of the excavation). Temporary excavation slopes within fine-grained fill soils should not exceed slopes of 1H:1V and should be limited to maximum depths of 15 feet. Actual slopes used during construction should be determined on a case-by-case basis. Shoring or foundation underpinning may be required in areas where sloping excavations cannot be employed due to space constraints and said excavation risks undermining bearing of adjacent foundations.

Shoring Deflection. Numerous studies have shown that shoring pressures are directly related to lateral movement of the shoring. An average lateral deflection at the top of a wall of approximately 1/1000 of the wall height should be adequate to mobilize the internal soil strength, thereby reducing the total lateral pressure to a semi-active state of stress (conventional design approach). With this level of deflection, the stress is distributed in a roughly parabolic shape, normally approximated as a rectangle. If lateral deflections are allowed to increase to within the range of 1/150 to 1/75 of the wall height, the pressure distribution starts to become triangular, with the greatest stress at the bottom of the wall. Quality construction procedures usually result in shoring deflections less than this.

Vertical deflections (settlements) immediately behind the wall may approach 2 to 3 inches, with settlements dissipating further from the wall. This assumes that good construction procedures are used. If unfilled voids are left behind the wall, or if walls are allowed to slough or cave before lagging is installed, the settlements can be far greater.

Design Shoring Pressures. We anticipate that cantilever shoring will be used. Cantilever shoring should be designed for a lateral earth pressure derived from an equivalent fluid weight of 31 pcf. This shoring pressure represents our best estimate of actual pressures that may develop against the shoring and does not contain a factor of safety. Adequate factors of safety must be incorporated in the design method.

With a cantilever design, it is not possible to totally eliminate deflections behind the wall. The design pressures presented are intended to limit the vertical deflection behind the wall to less than 1 inch (assuming a high level of care in construction). This deflection could be manifested as settlement or minor distress to the sidewalk but if proper care is exercised in construction, the adjacent street should not be noticeably affected.

These design pressures do not include seismic effects due to the low probability of a major seismic event occurring during the relatively short construction period.

Soldier Piles. Soldier piles must be designed for bending, vertical loads, and for passive kick out at the pile toe. Toe kick-out can be resisted by passive pressure against the base of the pile. For a horizontal ground slope at the base of the wall, passive pressures may be designed as a 500 pcf equivalent fluid weight. In the case of isolated soldier piles (center to center spacing greater than three pile diameters), these pressures may be applied to a width equal to three pile diameters. This pressure is our best estimate of actual pressures that can be developed and does not contain a factor of safety. We recommend using a safety factor of at least 1.5 in design against kick out.

Lagging. Lagging is not normally structurally designed, and thicknesses are usually based on empirical values. Soil arching at the soldier pile will substantially reduce stresses on the lagging. If lagging is to be designed, we recommend that design pressures be the same as those already presented, except that, due to arching, no stress need be included on the portion of lagging that is within one pile diameter of the soldier piles.

The permanent groundwater at this site probably will not affect shoring design. However, there is a possibility of perched water in some locations. If perched water occurs, the flows are expected to be minor and should be easily accommodated during construction.

6.2 Underpinning

It is anticipated that portions of the proposed excavations will be located adjacent to buildings that will not be demolished prior to excavation. In areas where the planned excavation encroaches on a zone extending down from building foundations at a 1H:1V slope, temporary building support is recommended. The most likely approach to provide temporary building support would be through the use of driven piling.

Allowable Pile Axial Capacity. We have developed allowable axial pile capacities for HP 14X89 piles (a commonly available pile). We anticipate that underpinning piles would be installed a minimum of 10 feet below the bottom of the proposed excavation. The following table indicates the allowable underpinning pile capacities for the design pile.

Allowable HP 14X89 Axial Pile Capacity

Embedment Depth Below Excavation	Allowable Axial Capacity
10 feet	29 tons
15 feet	35 tons
20 feet	38 tons
25 feet	40 tons
30 feet	42 tons
35 feet	45 tons

Pile Installation. Due to the proximity to existing structures, it is likely that a vibratory hammer would be used to install the piles. It is possible that the presence of obstructions in the fill could require the subsequent use of a driving hammer. Actual driving criteria would be based upon pile section, hammer type and allowable axial capacity, etc. Once these parameters have been established, Hart Crowser will establish pile driving criteria.

Group Effect. Reduction of axial pile capacity as a result of group effect can be ignored if center-to-center pile spacing is equal to, or in excess of three pile diameters.

6.3 Fills

Excavation backfill should be installed in horizontal lifts not exceeding 12 inches in thickness (loose - prior to compaction), and should be compacted to at least 90 percent of the maximum dry density. The maximum dry densities should be determined in accordance with ASTM D 1557.

During dry weather, structural fills may consist of virtually any relatively well-graded soil that is free of debris, organic matter, and high percentages of clay or clay lumps that can be compacted to the preceding specifications. However, if excess moisture causes the fill to pump or weave, those areas should be dried and re-compacted, or removed and backfilled with compacted granular fill. In order to achieve adequate compaction during wet weather or if proper moisture content cannot be achieved by drying, we recommend that fills consist of well-graded granular soils (sand or sand and gravel) that do not contain more than 5 percent material by weight passing the No. 200 sieve. In addition, it is usually desirable to limit this material to a maximum 6 inches in diameter for ease of compaction and future installation of utilities.

If excess soil moisture is present in potential fill soils, soil drying via aeration should be considered. Soils can commonly be dried by discing and turning in order to evaporate excess moisture. Soil drying in this manner is generally only possible during extended periods of warm dry weather.

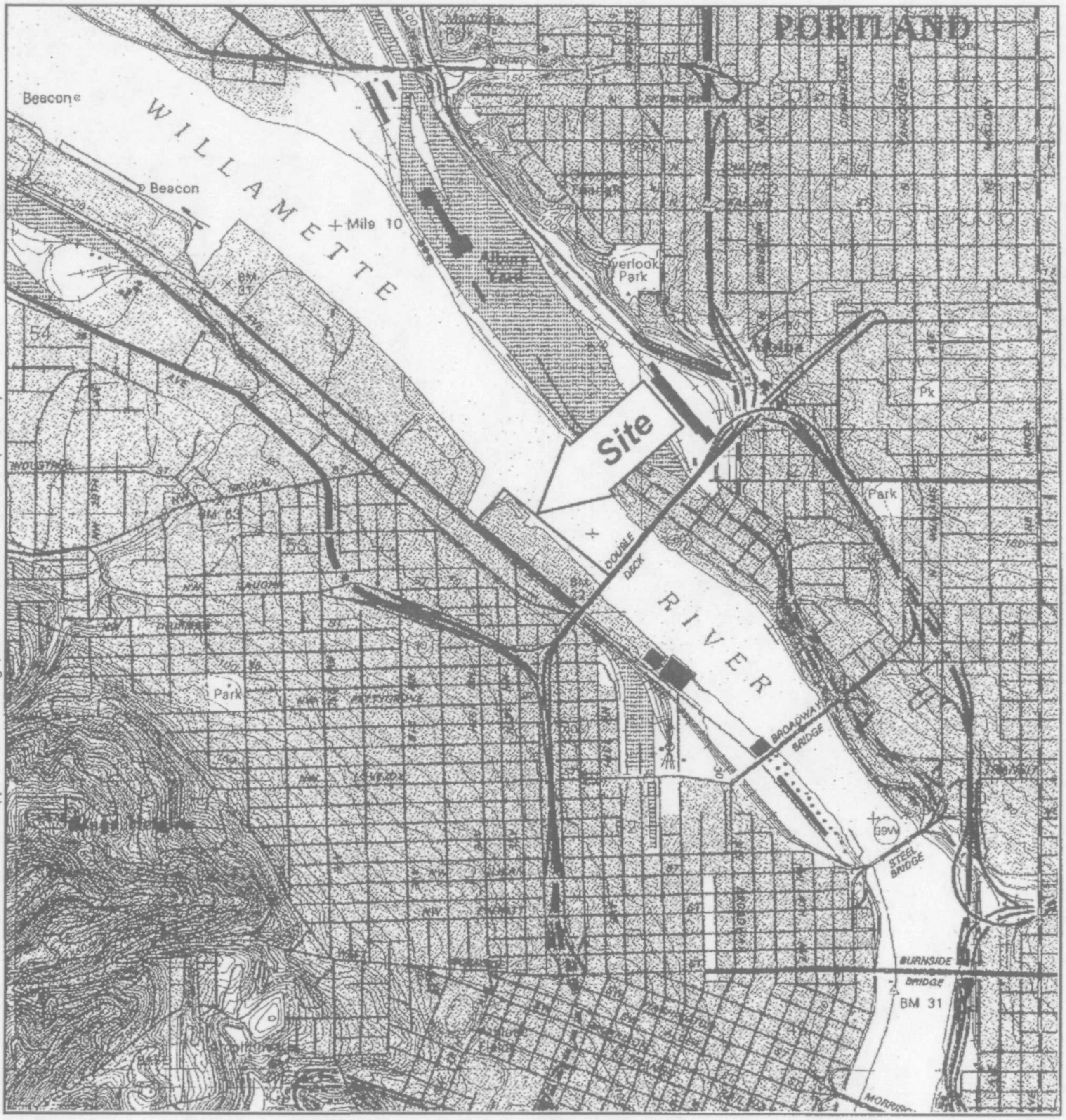
Quality Control During Fill Placement. To reduce the potential for long term fill related settlement issues, all fill and backfill should be observed and tested on a regular basis during construction. Observation and testing should be conducted to determine if compaction/density levels consistent with project plans and specifications are being achieved. Placement and compaction techniques as well as density testing should ideally be conducted on a lift-by-lift basis. This would usually entail at least one site visit per day by a soils inspector during rough grading operations.

7.0 CLOSING

This report presented Hart Crowser's geotechnical engineering evaluation and recommendations for the proposed project. We trust that this report meets your needs. If you have any questions, or if we can be of further assistance, please call. We look forward to working with you in the future.

Site Location Map
Port of Portland
Terminal 1

F:\DATA\Jobs\Port of Portland\15230 Term 1 Support\Rem Act Design, -02\Geotechnical\1523002 01 (Site Loc)



Note: Base map prepared from the USGS 7.5-minute quadrangle of Portland, OR dated 1990.

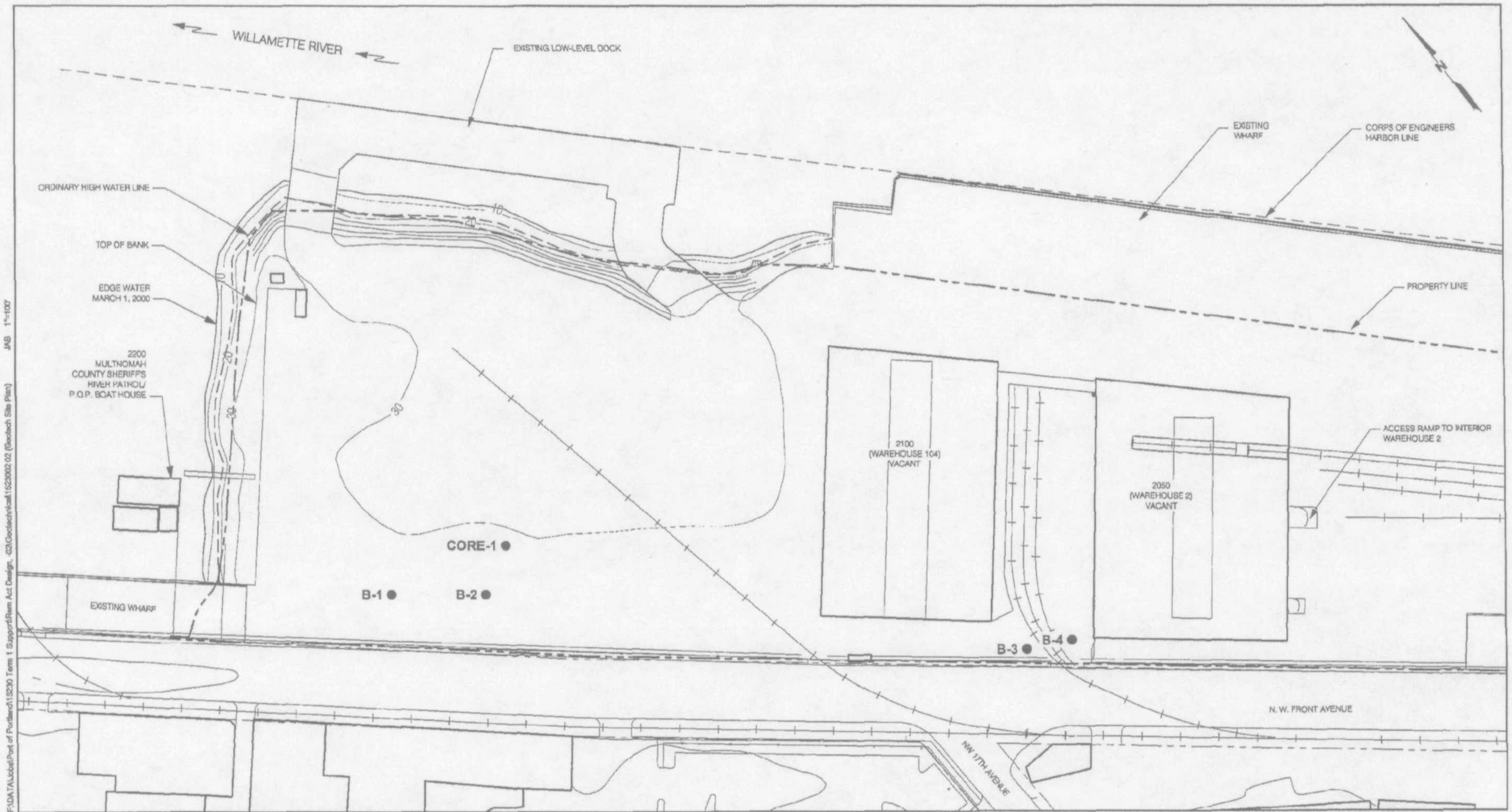
0 2,000 4,000

Scale in Feet
Contour Interval 10 Feet

N

HARTCROWSER
15230-02 **4/02**
Figure 1

Site Plan
Port of Portland
Terminal 1



Note: Base map prepared from an AutoCAD files provided by the City of Portland and the Port of Portland.

Legend:

B-1 ● Geotechnical Boring Location and Number

0 100 200
Approximate Scale in Feet

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Figure 2

APPENDIX A
FIELD EXPLORATIONS AND LABORATORY DATA

Key to Exploration Logs

Sample Descriptions

Classification of soils in this report is based on visual field and laboratory observations which include density/consistency, moisture condition, and grain size, and should not be construed to imply field nor laboratory testing unless presented herein. Visual-manual classification methods of ASTM D 2488 were used as an identification guide.

Soil descriptions consist of the following:
Density/consistency, moisture, color, minor constituents, MAJOR CONSTITUENT with additional remarks.

Density/Consistency

Soil density/consistency in borings is related primarily to the Standard Penetration Resistance. Soil density/consistency in test pits and push probe explorations is estimated based on visual observation and is presented parenthetically on test pit and push probe exploration logs.

SAND and GRAVEL Density	Standard Penetration Resistance in Blows/Foot
Very loose	0 - 4
Loose	4 - 10
Medium dense	10 - 30
Dense	30 - 50
Very dense	>50

SILT or CLAY Density	Standard Penetration Resistance in Blows/Foot	Approximate Shear Strength in TSF
Very soft	0 - 2	<0.125
Soft	2 - 4	0.125 - 0.25
Medium stiff	4 - 8	0.25 - 0.5
Stiff	8 - 15	0.5 - 1.0
Very Stiff	15 - 30	1.0 - 2.0
Hard	>30	>2.0

Moisture

Dry	Little perceptible moisture.
Damp	Some perceptible moisture, probably below optimum.
Moist	Probably near optimum moisture content.
Wet	Much perceptible moisture, probably above optimum.

Minor Constituents

Minor Constituents	Estimated Percentage
Not identified in description	0 - 5
Slightly (clayey, silty, etc.)	5 - 12
Clayey, silty, sandy, gravelly	12 - 30
Very (clayey, silty, etc.)	30 - 50

Legends

Sampling Symbols

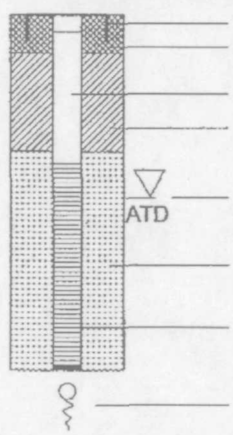
BORING SYMBOLS

- Split Spoon
- Tube (Shelby, Push Probe)
- Cuttings
- Core Run
- * No Sample Recovery

TEST PIT SOIL SAMPLES

- Grab (Jar)
- Bag
- Shelby Tube

Groundwater Observations and Monitoring Well Construction



Test Symbols

GS	Grain Size Classification
K	Permeability
AL	Atterberg Limits

Water Content in Percent
- Liquid Limit
- Natural
- Plastic Limit

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Figure A-1

Boring Log B-1

Soil Descriptions

9" Asphalt concrete.

Stiff, dry, brown, sandy, gravelly SILT (FILL).

Medium dense, dry, brown, silty SAND (FILL) with some rounded gravel.

Soft, damp, brown and black, sandy SILT (FILL) with some wood chips.

Becomes gray and black with a petroleum-like odor.

Medium stiff, moist to wet, gray mottled, fine, sandy SILT.

Basalt chunk lodged in shoe.

Becomes tannish-brown.

Lenses of gravel and sandy silt below 32'.

Rock lodged in shoe.

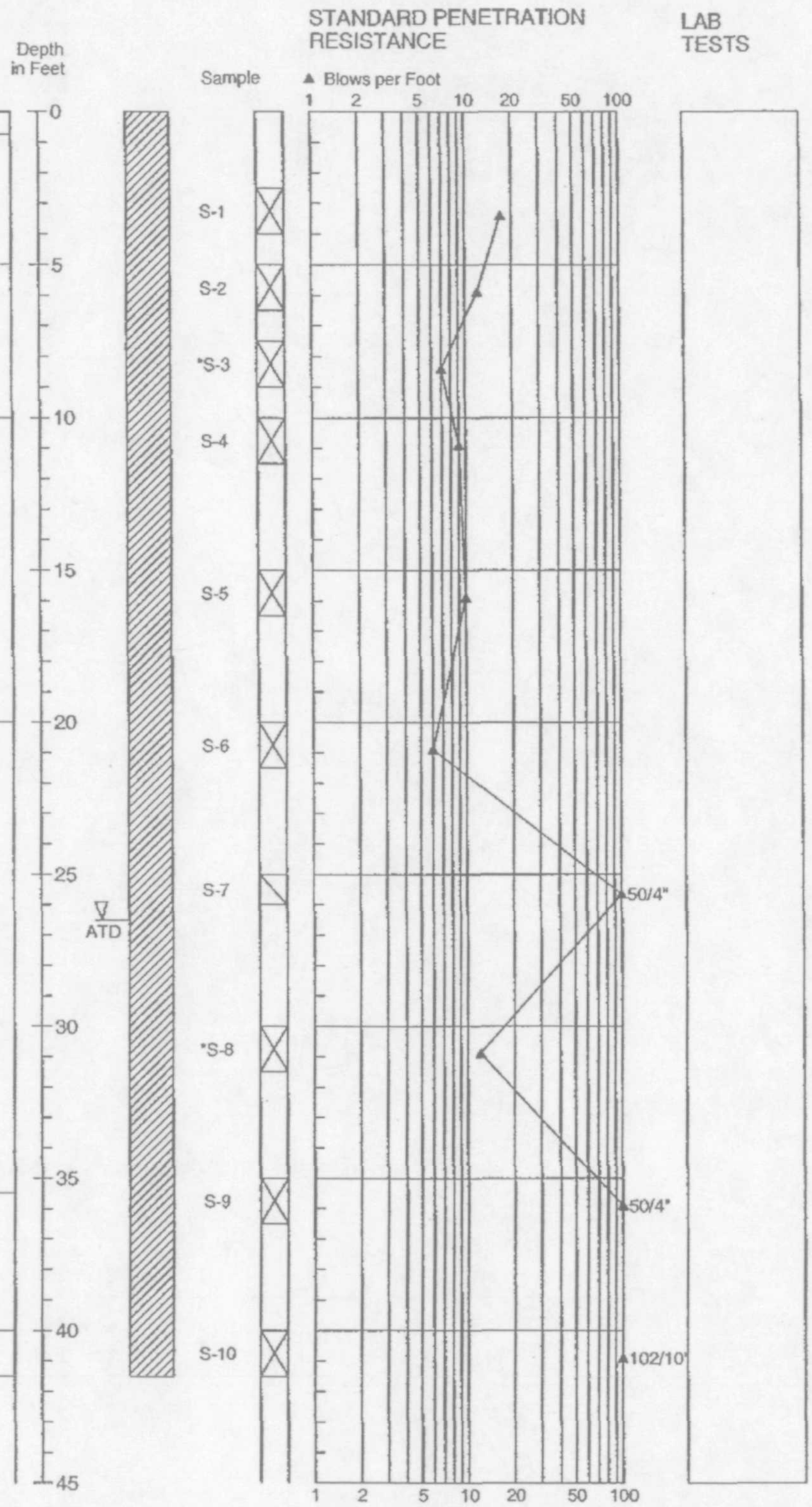
Dense, wet, gray, coarse SAND.

Gravel lenses.

Very dense, wet, gray, coarse SAND and GRAVEL.

Bottom of Boring at 41.5 Feet.

Completed 03/25/02.



BORING LOG WITH BLOW COUNTS B_1523-1.GPJ HC_CORP.GDT 4/23/02

1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. Groundwater level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.

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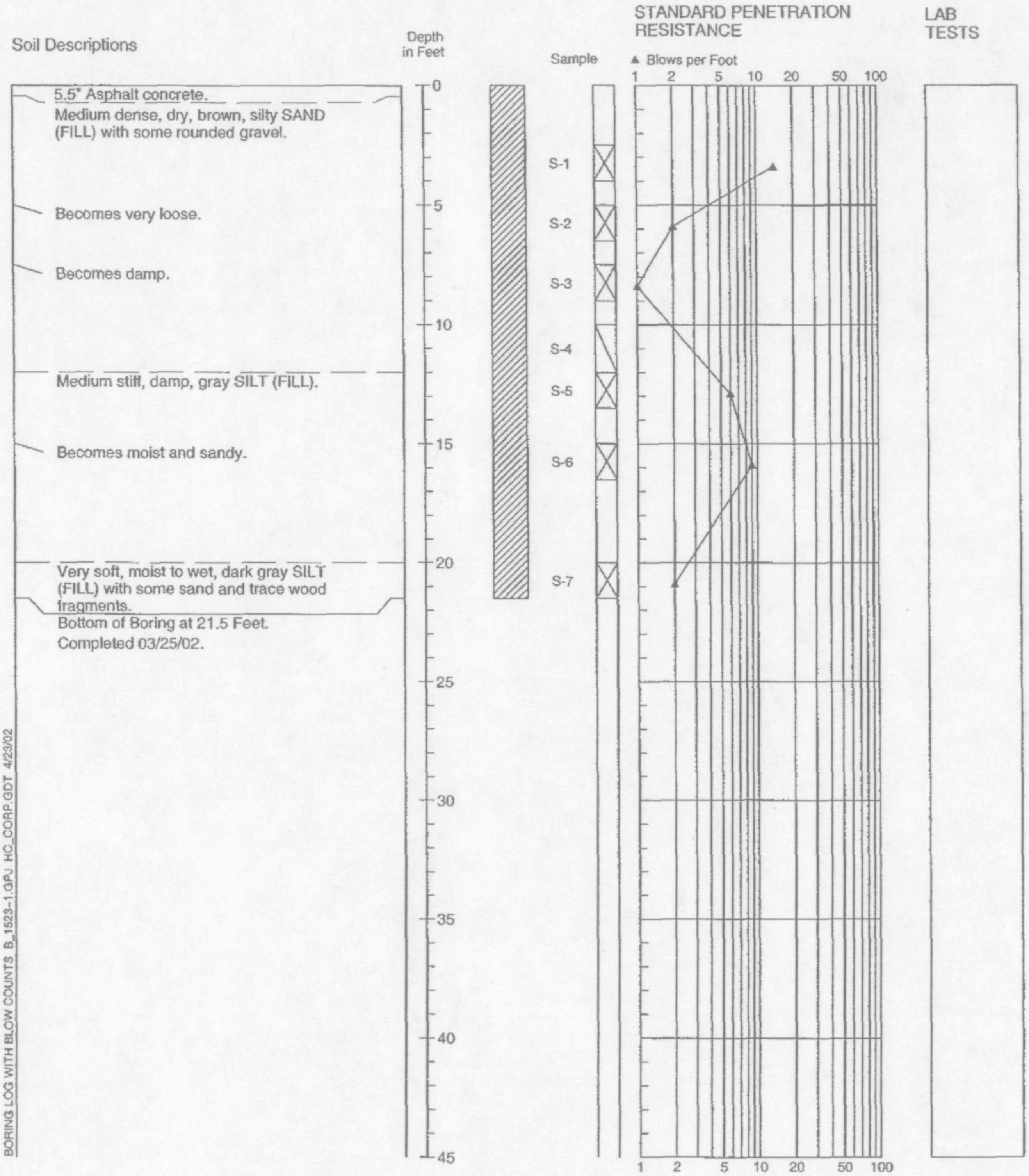
Figure A-2

POPT1S601023

Boring Log B-2



1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. Groundwater level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.

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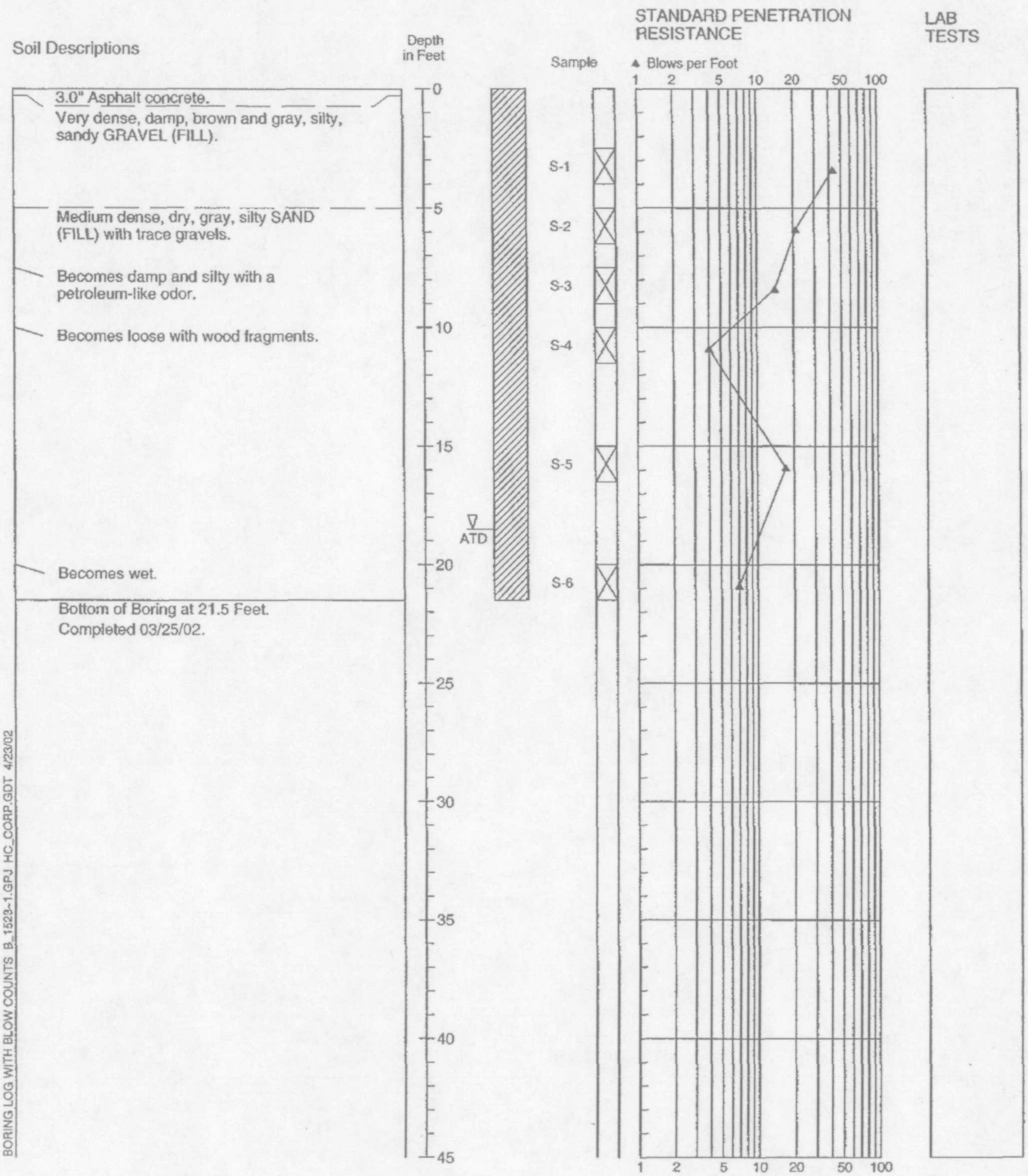
Figure A-3

POPT1S601024

Boring Log B-3



1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. Groundwater level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.

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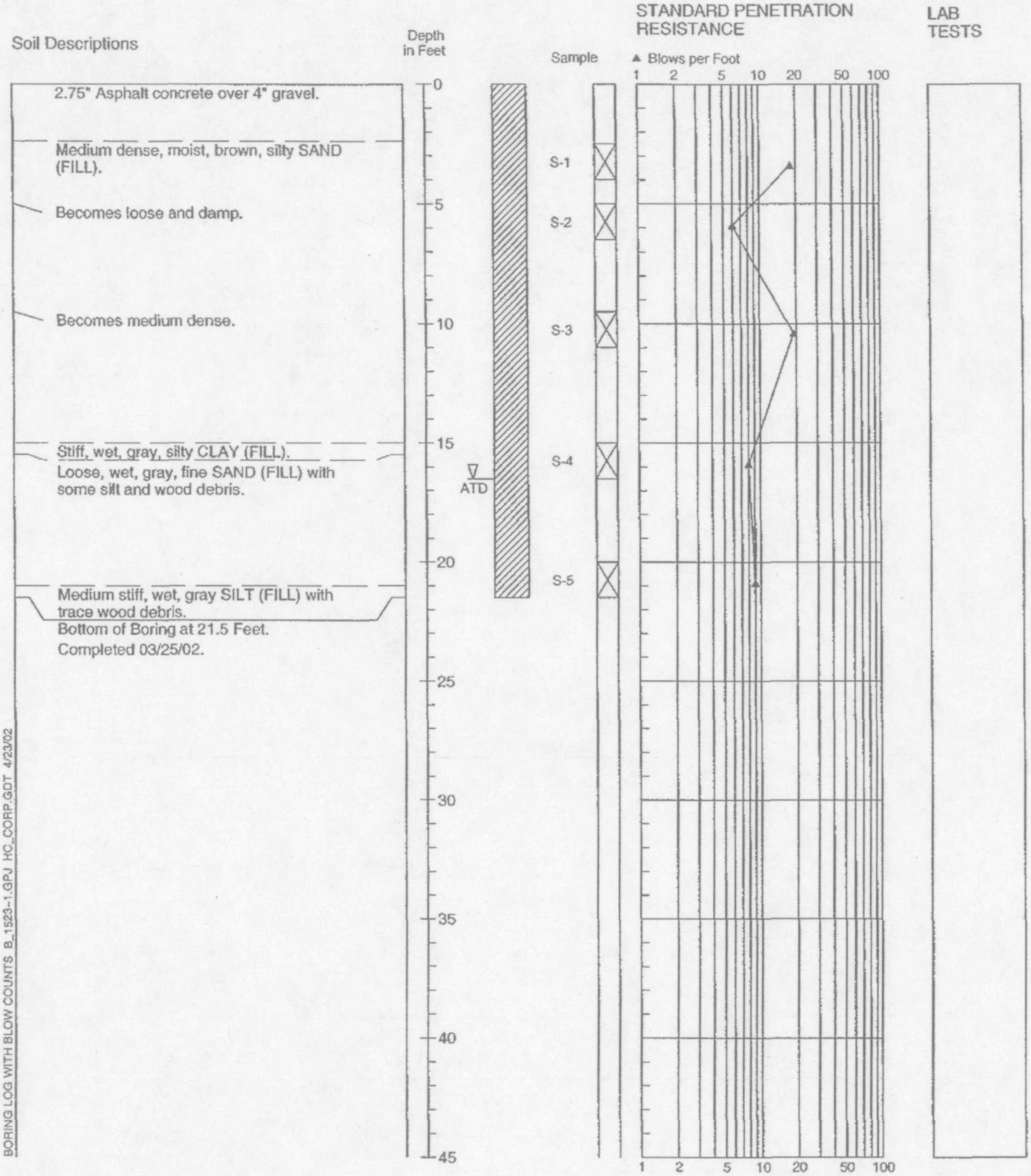
15230-01 3/02

Figure A-4

POPT1S601025

Boring Log B-4



1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. Groundwater level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.

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15230-01 *3/02*

Figure A-5

POPT1S601026

Moisture Content

Job Terminal 1 Project Tested by GWK Date 4/1/02
Job No. 15230-1 Calculated by Checked by

Exploration No.	B-1	—	—	—	—	—
Sample No.						
Sample Depth (ft)	2.5'	5.0	10	15	20	26
Container No.	10	11	12	13	14	15
Wt. Tare	3.00	—	—	—	3.01	—
Wt. Wet Soil & Tare	104.57	50.18	86.55	149.84	184.25	130.77
Wt. Dry Soil & Tare	95.89	46.39	65.45	117.19	131.74	97.19
Wt. Water						
Wt. Dry Soil						
Moisture Content %	9	9	33	29	41	36
Comments		used all sample				

Exploration No.	B-1	Terminal	B-2	—	—	—
Sample No.						
Sample Depth (ft)	40	35	2.5	5.0	7.5	12-13.5'
Container No.	16	17	18	19	20	21
Wt. Tare	3.01	—	—	—	—	—
Wt. Wet Soil & Tare	143.43	158.60	68.11	101.64	76.54	101.37
Wt. Dry Soil & Tare	126.22	126.73	62.81	92.35	68.62	70.51
Wt. Water						
Wt. Dry Soil						
Moisture Content %	14	26	9	10	12	46
Comments					used all sample	

Exploration No.	B-2	—	B-3	—	—	—
Sample No.						
Sample Depth (ft)	15	20	2.5	5.0	7.5	10
Container No.	22	23	R-4	24	25	R-6
Wt. Tare	3.01	—	40.19	3.01	—	84.07
Wt. Wet Soil & Tare	148.13	119.76	704.71	84.76	160.67	348.82
Wt. Dry Soil & Tare	111.20	80.32	666.72	77.09	139.67	292.80
Wt. Water						
Wt. Dry Soil						
Moisture Content %	34	51	7	10	15	28
Comments			used all sample			used all sample

Remarks

Note: Calculate and record moisture content to nearest percent.

$$\text{Moisture Content} = \frac{\text{Wt. Water}}{\text{Wt. Dry Soil}} = \frac{\text{Wt. Wet - Wt. Dry}}{\text{Wt. Dry}}$$

HART-CROWSER & associates Inc.
Form L105-81

POPT1S601027

Moisture Content

Job Terminal I Project Tested by GWK Date 4/1/02
Job No. 15230-1 Calculated by ______ Checked by ______

Exploration No.	B-3		B-4		Terminal I	
Sample No.					Cuttings	
Sample Depth (ft)	15	20	15	20	5' to 7.5'	
Container No.	26	27	28	29	30	
Wt. Tare	3.02	—	—	—	—	
Wt. Wet Soil & Tare	45.28	94.32	90.45	122.21	83.28	
Wt. Dry Soil & Tare	25.57	70.70	67.87	86.64	76.38	
Wt. Water						
Wt. Dry Soil						
Moisture Content %	87	35	36	43	9	
Comments	wood	used all sample				

Exploration No.						
Sample No.						
Sample Depth (ft)						
Container No.						
Wt. Tare						
Wt. Wet Soil & Tare						
Wt. Dry Soil & Tare						
Wt. Water						
Wt. Dry Soil						
Moisture Content %						
Comments						

Exploration No.						
Sample No.						
Sample Depth (ft)						
Container No.						
Wt. Tare						
Wt. Wet Soil & Tare						
Wt. Dry Soil & Tare						
Wt. Water						
Wt. Dry Soil						
Moisture Content %						
Comments						

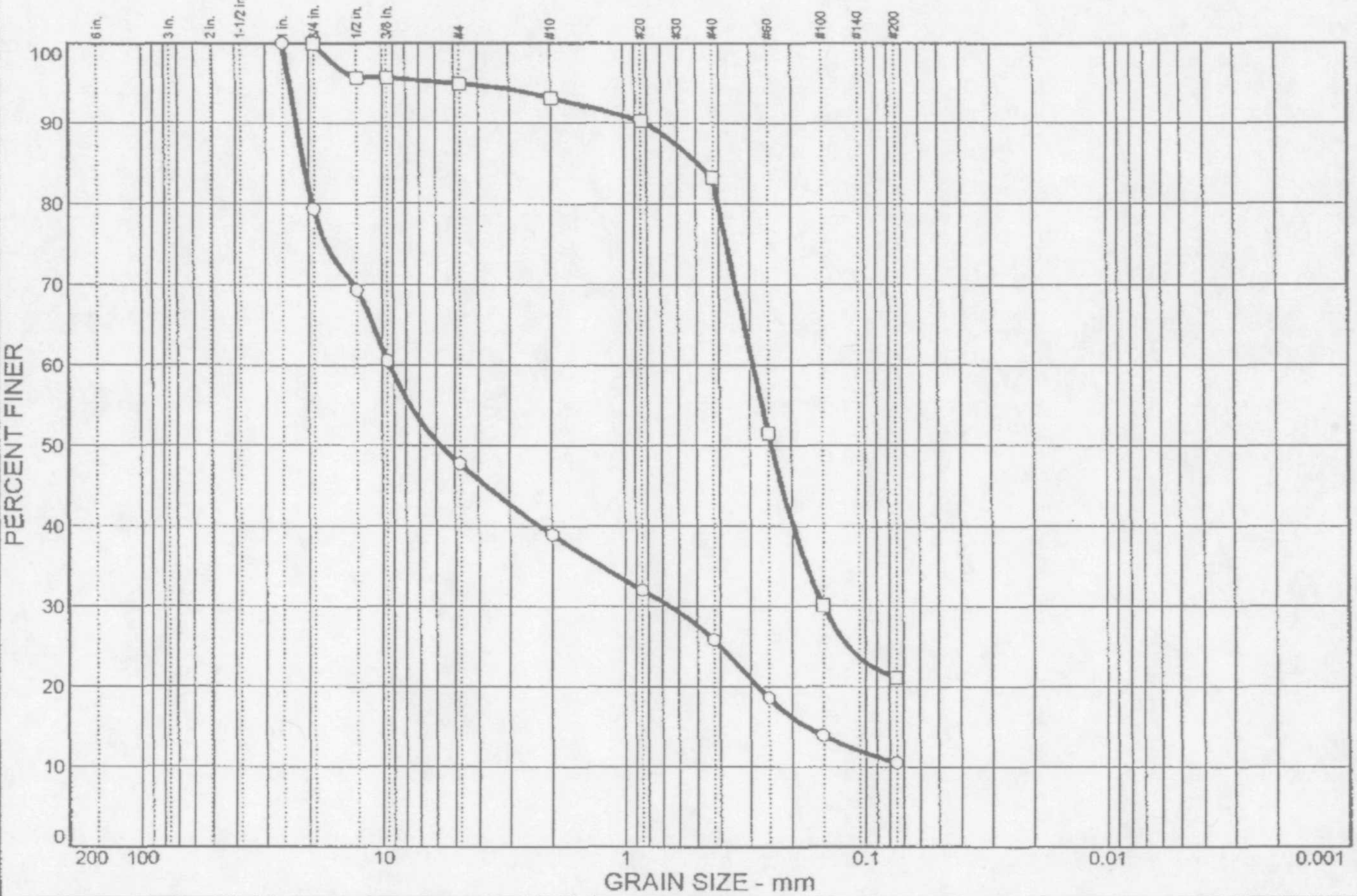
Remarks ______

Note: Calculate and record moisture content to nearest percent.

$$\text{Moisture Content} = \frac{\text{Wt. Water}}{\text{Wt. Dry Soil}} = \frac{\text{Wt. Wet} - \text{Wt. Dry}}{\text{Wt. Dry}}$$

HART-CROWSER & associates inc.
Form L105-81

PARTICLE SIZE DISTRIBUTION TEST REPORT



	% + 3"	% GRAVEL		% SAND			% FINES	
		CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
○	0.0	20.7	31.6	8.8	13.1	15.3	10.5	
□	0.0	0.0	5.1	1.8	9.9	62.1	21.1	

╳	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○			21.0	9.37	5.72	0.653	0.174			
□			0.490	0.290	0.243	0.149				

MATERIAL DESCRIPTION	USCS	NAT. MOIST.
○ Slightly silty, very sandy GRAVEL	GP-GM	7%
□ Slightly gravelly, silty, medium to fine SAND	SM	28%

Remarks:

- ○ Used entire sample
- □ Some "product" odor, used entire sample, much woody material

Project: Terminal 1 Project

Client: Port of Portland

○ **Source:** B-3 **Elev./Depth:** 2.5'

□ **Source:** B-3 **Elev./Depth:** 10'

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15230-01 4/2/2002

Figure No.

GRAIN SIZE DISTRIBUTION TEST DATA

Client: Port of Portland
Project: Terminal 1 Project
Project Number: 15230-01

Sample Data

Source: B-3
Sample No.:
Elev. or Depth: 2.5'
Sample Length (in./cm.):
Location:
Description: Slightly silty, very sandy GRAVEL
Liquid Limit:
Plasticity Index:
Natural Moisture: 7%
USCS Classification: GP-GM
Testing Remarks: Used entire sample

Mechanical Analysis Data

	Initial	After wash
Dry sample and tare=	666.72	606.53
Tare =	90.19	90.19
Dry sample weight =	576.53	516.34

Minus #200 from wash= 10.4 %
Tare for cumulative weight retained= .00

Sieve	Cumul. Wt. retained	Percent finer
1 inch	0.00	100.0
.75 inch	119.38	79.3
.5 inch	176.92	69.3
.375 inch	227.93	60.5
# 4	301.68	47.7
# 10	352.16	38.9
# 20	391.85	32.0
# 40	427.79	25.8
# 60	469.17	18.6
# 100	496.14	13.9
# 200	516.22	10.5

Fractional Components

Gravel/Sand based on #4
Sand/Fines based on #200
% + 3" = % GRAVEL = 52.3 (% coarse = 20.7 % fine = 31.6)
% SAND = 37.2 (% coarse = 8.8 % medium = 13.1 % fine = 15.3)
% FINES = 10.5

D_{85}= 20.97 D_{60}= 9.37 D_{50}= 5.72
D_{30}= 0.65 D_{15}= 0.17

GRAIN SIZE DISTRIBUTION TEST DATA

Client: Port of Portland
Project: Terminal 1 Project
Project Number: 15230-01

Sample Data

Source: B-3
Sample No.:
Elev. or Depth: 10' Sample Length (in./cm.):
Location:
Description: Slightly gravelly, silty, medium to fine SAND
Liquid Limit: Plasticity Index:
Natural Moisture: 28% USCS Classification: SM
Testing Remarks: Some "product" odor, used entire sample, much woody material

Mechanical Analysis Data

	Initial	After wash
Dry sample and tare=	292.80	250.09
Tare =	89.07	89.07
Dry sample weight =	203.73	161.02

Minus #200 from wash= 21.0 %
Tare for cumulative weight retained= .00

Sieve	Cumul. Wt. retained	Percent finer
.75 inch	0.00	100.0
.5 inch	8.69	95.7
.375 inch	8.69	95.7
# 4	10.44	94.9
# 10	14.10	93.1
# 20	19.73	90.3
# 40	34.17	83.2
# 60	98.57	51.6
# 100	142.33	30.1
# 200	160.76	21.1

Fractional Components

Gravel/Sand based on #4
Sand/Fines based on #200
% + 3" = % GRAVEL = 5.1 (% coarse = % fine = 5.1)
% SAND = 73.8 (% coarse = 1.8 % medium = 9.9 % fine = 62.1)
% FINES = 21.1

D_{85}= 0.49 D_{60}= 0.29 D_{50}= 0.24
D_{30}= 0.15